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DESIGN
OF A
REINFORCED CONCRETE ORE DOCK

BY

HARRY NICHOLL STARKEY

A THESIS SUBMITTED FOR
THE DEGREE OF CIVIL ENGINEER

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1913

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DESIGN OF A REINFORCED CONCRETE ORE DOCK

The author became interested in the design of ore docks while employed as draftsman in the office of the Chief Engineer of the Canadian Northern Railway at Duluth, Minn. During the summer of 1910 the Company made calculations in regard to the cost and design of a dock which they expected to construct the next year; and the author resolved then to make a study of the subject. He went back to the University in the fall of 1910, and took with him all the data at hand in regard to the conditions for which the dock was to be designed. Financial embarrassment made it necessary for him to do outside work during the school year and also to accept a position six weeks before the end of the term, with the result that the thesis which he endeavored to write upon the subject was not finished in the spring of 1911. He has written it almost entirely during his evenings at home since that time. In all he has put 380 hours upon the work of designing, detailing and tracing, and in the space allotted he is only able to show a few of the principal calculations which were made in determining the size and style of the many details which are shown on the plans.

An effort has been made to design a dock which

could be built with a minimum cost, would have the required strength and durability, and would be practical in construction. While many of the methods of stress determination and design are partially original, the author has based them largely upon the principles which he has absorbed during his course at the University of Wisconsin. Use was made of the tables and ideas given in Turneaure and Maurer's text book entitled "Principles of Concrete Construction", and from Taylor's "Concrete, Plain and Reinforced". A large amount of reading was done in connection with the topic from the current literature and from the treatise on concrete by Reid, Marsh and several others. In personal interviews, Mr. Clark, Chief Engineer of the Duluth & Iron Range Railroad, Mr. Johnson, of the Wisconsin Bridge and Iron Company, and Mr. Dickerson, the inventor of the Dickerson ore pocket, each gave the author help from their experience as ore dock designers.

The following statement in regard to the dimensions of the dock to be designed was given by Mr. H. T. Hazen, who was then Chief Engineer of the Canadian Northern Railway at Duluth.

"The dock would be 1080 feet long, about 60 feet wide and 65 feet high, from base of rail to mean water

level. It would contain 180 ore pockets, 90 on each side, and each of the pockets would have a capacity of 190 tons. On its deck would be four railway tracks and five sidewalks for the accommodation of the men emptying cars. The lake bottom at the site of the dock is sand and the water varies in depth from 25 to 40 feet."

By way of general information we may say that an ore dock is a long, high trestle, extending out into a navigable body of water, which accommodates railroad tracks on its deck and has "pockets" under its deck into which the ore is dumped directly from the bottoms of the cars, and from the lower part of which the ore may flow thru chutes into the holds of the boats moored at the side of the dock.

There are a number of these docks at the ports on the Great Lakes. The ore trains run continuously night and day from the mining towns, and running out on the docks dump their loads of ore into the pockets; where it is stored until some ship, bound for down the lakes, is ready to receive it.

Heretofore most of these docks have been constructed of timber entirely, and have proved to have a life period of ten years. Within the last two or three years,

however, an effort has been made to construct docks which will have a longer life of usefulness. The Duluth & Iron Range Railway has built two docks of structural steel at Two Harbors, Minn. The Great Northern Railway has built a dock of steel on concrete piers at Superior, Wisconsin, and the Wisconsin Bridge & Iron Company have recently completed the construction of a dock at Marquette, Michigan, which is built entirely of reinforced concrete.

Many ideas have been formed by visiting some of these structures, but it will be noticed that the dock designed in this thesis is entirely different from any other yet devised, excepting, of course, in such standard characteristics as general shape of pocket, height and distance between chutes, track arrangements, etc.

In the design the following assumptions were made:

WORKING STRESSES RECOMMENDED BY THE JOINT COMMITTEE 1909, FOR CONCRETE HAVING AN ULTIMATE COMPRESSIVE STRENGTH OF 2000 lbs. per sq. in. IN 28 DAYS.

$f_s =$ 16000 lbs. per sq. in.
$f_c =$ 650 " " " " for beams
$n =$ 15

$f_c =$ 450 " " " " for columns

RECOMMENDED BY THE AMERICAN BRIDGE CO. FOR STEEL BRIDGES

$$I = S\left(\frac{300}{L + 300}\right)$$

f_s 16000 lbs. per sq. in.

$s = 10000$ " " " "

ALSO THE FOLLOWING WORKING STRESSES WERE ASSUMED-

$V = 40$ lbs. per sq. in. (for concrete not reinforced)
$v = 120$ " " " " (for concrete well reinforced)
$U = 60$ " " " " (for round bars)
$u = 150$ " " " " (for deformed bars)

$W_o = 160$ " " " "
$W_c = 100$ " " " "

$C_s = .0083$ dollars per cu. in. of steel
$C_c = .000108$ " " " " " concrete

Allowable load per pile 20 tons.

Percentage of train load considered as longitudinal thrust 20.

Angle of repose of ore 30 degrees.

LEGEND

f_s = working tensile strength of steel reinforcement.

f_c = " compressive strength of concrete.

n = relation of coefficients of elasticity of steel and concrete.

V = working shearing strength for concrete not reinforced.

v = " " " " " well "

U = bond stress allowed between concrete and round steel bars.

u = bond stress allowed between concrete and deformed steel bars.

W_o = weight of iron ore.

W_c = " " concrete.

C_s = cost of putting in steel.

C_c = " " " " concrete.

As far as possible the thesis is put under these two headings: Determination of Stresses, and Design of Members. The determination of stresses will first be considered.

DETERMINATION OF STRESSES

1st

Stresses due to trains loaded with iron ore at top of dock.

The rails will rest on I beams spanning the 12 foot pockets and supported at ends by the lateral walls of the pockets; and the stresses in these I beams will be due to the locomotive loads which are given in the figure #1.

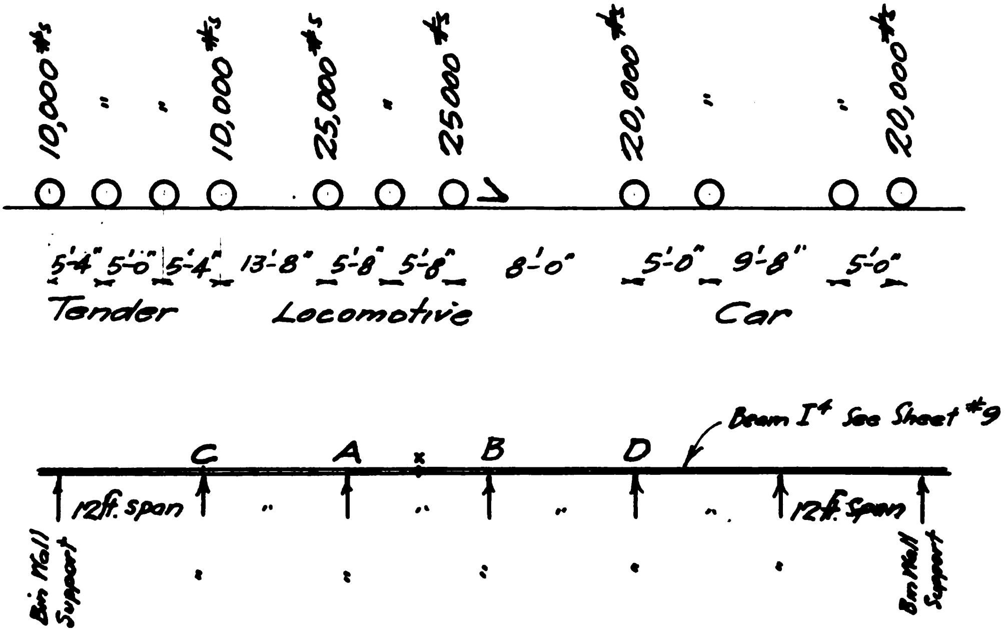


FIG. 1.

By trial it is found that the maximum moment will occur on any span as AB at the center when wheel #6 is at the center of the span. The three loads #5, #6 and #7 will then be on span AB and since they will then be symmetrically located with respect to the center of the span one-half their total load (37,500#) is the reaction at A and B, and the moment is 37500 x 6 - 25000 x 5.66 = 83,300 ft. lbs. or 1,000,000 in. lbs.

Similarly, the maximum shear will occur at say A when the wheel #5 is just at the right of and #6 and #7 are respectively 68 in. and 136 in. from A. Then the shear at A will be, $25000 \quad \frac{2500 \times 8 + 25000 \times 76}{144} = 39,600.0\#$.

The maximum load which could be concentrated at a point as A will occur when wheel #6 is at A. Then #5 will be 68 in. to left and #7 will be 68 in. to right of A and the reaction due to #5 and #7 will be $2 \times \frac{25 \times 76}{144} =$ 26000#. Then the total load at A = 26000 + 25000 = 51000#.

$I = S\left(\frac{300}{L + 300}\right) = 51000 \times 0.96 = 49000\#$ (this is the stress to be added due to the impact of the train.)

Applying the impact formula to the moment and shear determined above we can derive the table of total loads as given below:

TABLE #1

	Live load	Impact	Total load
Max. load (concentrated)	51,000	49,000	100,000
" shear	39,600	38,000	77,600
" moment	83,300	80,000	163,300

2nd.

Ore pressure against side of bins.

The ore in bins will cause a stress due to its weight which will act in a vertical direction, and also a pressure in a horizontal direction on the sides of the

bins. The latter will be determined by three methods, which are taken respectively from Baker, Rankin and Turneaure. The results will be averaged to obtain working values. In each case the pressure is determined for various heights of a bank of ore which leans against a vertical wall, and the action line of the resultant of the pressure is assumed to act at one-third the distance from the bottom to the top of the bank.

(a) Explanation of the first method, from Baker. Baker assumes, Fig. 2, that if DB be a vertical wall against which ore is banked from D to B, and BO be the horizontal, and if the angle of repose of ore be laid off, as O B F and the angle D B F be bisected, the pressure which will act at a point 1/3 BD from B, will be equal to the resultant P of the weight of ore W in the area D B E when it is assumed to slide on the smooth surface BE.

Since we assume a section one foot wide,

$W = w/2 \times h \times h \times \tan 30°$

and, $P = W \times \tan 30°$

then, $P = 160/2 \times h^2 \times \tan 30°$

where, w = 160#/cu.ft. assumed weight of ore

W = total weight of ore in area DBE

P = pressure which acts against the wall DB at a point 1/3 DB from B.

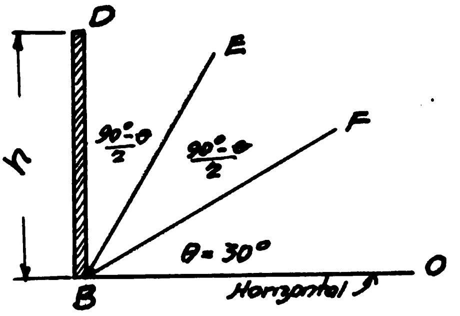


FIG 2

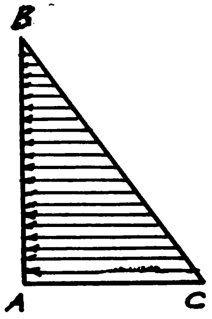


FIG 3

(b) Explanation of the second method, from Rankin. Rankin assumes that $P = Wh^2/2(1-\sin\theta/1+\sin\theta)$

(c) Explanation of the third method, from Turneaure. Turneaure assumes that earth pressure against a wall may be considered to act practically as a fluid of weight "p". After a due consideration of the values of "p" which Turneaure has taken for various earths, I have decided to use an equivalent weight for ore of p = 40.

Table #2 shows results obtained by each of the three methods, assuming heights of bank of ore as shown in column under "h". It also shows pressure on bottom foot of each of the depths of ore.

TABLE #2

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
h		1st	2nd	3rd		P	R
7.6	5.1	1530	1530	1160	4220	1400	369
8.6	5.8	2060	2060	1550	5650	1880	428
11.0	7.0	3220	3190	2420	8830	2940	533
13.0	8.6	4500	4450	3380	12330	4100	630
14.7	9.7	5750	5700	4320	15770	5250	716
16.9	11.3	7600	9240	5720	20870	6950	822
18.7	12.5	9350	9240	7000	25900	8540	912
20.6	13.9	11400	11100	8500	31000	10300	1000
22.7	15.0	13700	13500	10320	37520	12400	1080

Column #2 of the above table gives the distance of the point of application of pressure from top of bank; column #6 is the total of methods 1, 2 and 3; and column #7 gives the average. I assume that the pressure due to the bank of ore varies from top to bottom of the bank in a straight line relation, having a value of zero at the top and a maximum at the bottom. P is a summation of the pressure from zero to a maximum, and is given in Fig. #3 as the shaded area ABC, AB = h, the height of bank. The maximum pressure AC = R lbs. per sq. ft. Column #8 is equal to twice the average, or P/h x 2 = R. The values in column #8 were calculated from this formula, and they represent the actual pressure on the bottom foot of each of the banks of ore (h) given in column #1.

Curve (R) shows the values in the table plotted

with R as ordinates and (h) as abscissa. The equation of the curve is seen to be very closely 21 x R = 1000 x h, and the following table gives the values of R for each foot as calculated from this formula. Twelve R is given for convenience of reference as it is the total pressure on each section one foot high across the front of the bin.

TABLE #3

(1)	(2)	(3)
h	R	12R
1	47	571
2	95	1140
3	143	1730
4	190	2280
5	238	2860
6	285	3430
7	333	4000
8	381	4580
9	429	5150
10	475	5700
11	525	6310
12	572	6870
13	620	7450
14	667	8000
15	715	8600
16	763	9180
17	810	9740
18	858	10300
19	905	10800
20	954	11400
21	1000	12000
22	1035	12400
23	1095	13100

3rd

Stresses in columns supporting bins due to train load, ore load and weight of structure.

These forces are determined for all conditions of loading so that a column may be designed to support the structure. The weight of the structure will first be computed roughly and its center of gravity determined.

The parts making up the weight of the structure may be figured under three heads according to the position

of their center of gravity. Then the center of gravity of these combined weights is determined graphically on sheet #1. The weight of the wall at front of bin is as follows. See Fig. 4 for section considered.

See Sheets #9 & 10 for Location

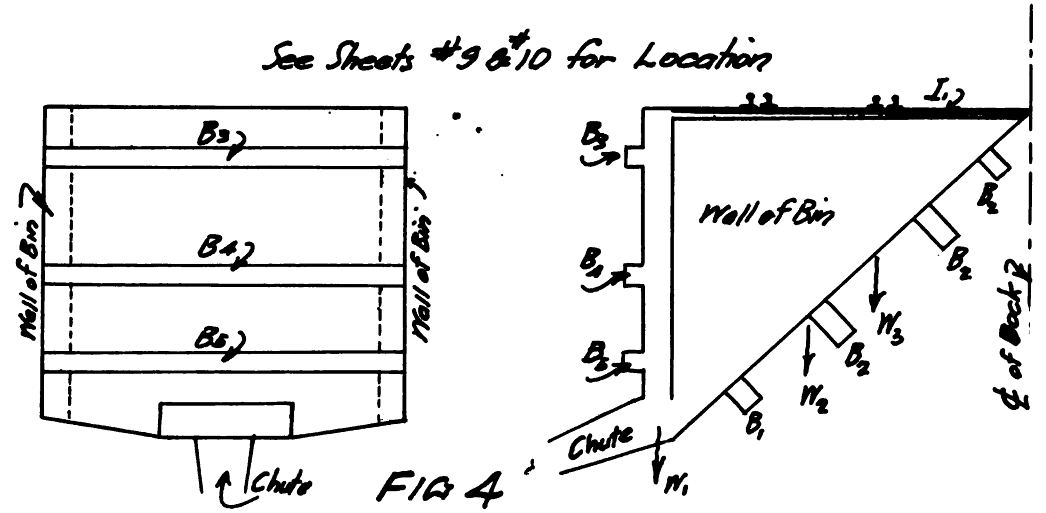


FIG 4

Volume of slab	$\frac{4.4 \times 23 \times 12 \times 144}{1728}$	101 cu. ft.
Volume of beam 5	$\frac{15 \times 10 \times 144}{1728}$	12.5 cu. ft.
Volume of beam 4	$\frac{13 \times 8 \times 144}{1728}$	8.7 cu. ft.
Volume of beam 3	$\frac{12 \times 6 \times 144}{1728}$	6.0 cu. ft.
	total vol.	128.2 cu. ft.

At 100 lbs. per cu. ft. this gives a weight of 12820# = W_1

The weight of the concrete partition wall of the bin is as follows. The total weight is roughly $\frac{23 \times 23 \times 1}{2} \times 100 = 26{,}600\# = W_2$ The other members of

the dock are symmetrical about a point half way between center and front of structure and therefore have their center of gravity at that point and will be considered together as follows:

½ member I (see sheet #9)	420
4 x wt. member I "	4550
wt. of sidewalk on deck	7630
wt. of 6" floor of hard maple at 50# per cu.ft.	
$\frac{6 \times 12 \times 23 \times 50}{12 \times \cos 45} =$	9780
wt. of 4 floor beams, B and B	
chord $\frac{15 \times 12 \times 144}{1728} = 14.95$	
two struts $\frac{15 \times 12 \times 36 \times 2}{1728} = 7.00$	
Total vol. = 29.47 cu.ft.	
Four of these at 100# per cu. ft. gives	11790
wt. of 24 ft. of 100# Rail	1600

	35770#=W_3

The action line of the total weight of ½ the structure which will bear on one column is the sum of these three, and is given on sheet #1.

On sheet #2 the resultant of the weights of the structure (75.190), ore 5 feet from the top of the bin (285.000), and a locomotive load on the outside track

(200.000) is determined.

On sheet #3 the resultant of the weights of the structure (75.190), train on the inside track, and ore piled up on the bottom of the bin (150.000). This I have called R_6

On sheet #4 the resultants of the weights of the structure (S), the bin full of ore (O), and locomotive loads on track (T & T) is determined. This I call R

On sheet #5 the resultant of the weight of the structure (S), and of a train on track (T) is determined. This is (R).

On sheet #6 the resultant of the weight of train on track (T), of the structure (S), and of the bin full of ore (O) 465,000 is determined and is called R_7

On sheet #7 the resultants of the weights of structure, train on track (T_2), and of bin full of ore (O) is determined and is called (R_8).

The resultants (R R R R R) as determined above are plotted on sheet #8 and are enumerated in the table #4 together with the required area of column cross-section to care for the respective loads, considering an allowable working stress of 450# per sq. in.

At any one time only one of these resultants, (R, R_6 R_7 R_8 or R_9) can act; and a column must be designed

which will care for any one of them, then it will care for all, as only one can act at one time. A great many sections of column were assumed with the object of finding one in which the action line of each resultant ($R, R_6 R_7 R_8$ or R_9) would pass thru a point which would be the center of gravity of a part of the section equal to the area required for that resultant as given in the table #4. The shaded areas on sheet #8 are the ones required for the respective resultants, and as each lies entirely within the column section as designed that section is the one to carry the loads without eccentricity.

TABLE #4

(1) Resultant	(2) W (#)	(3) Area Req'd.(sq.ft.)
R	75,190	1.13
R	559,190	8.62
R	425,190	6.55
R	740,190	11.40
R	740,190	11.40
R	940,190	14.50

4th

Stresses due to wind pressure on side of dock.

Considering a wind pressure of 40# per sq. ft. we would roughly have a total wind pressure on side of

dock tending to overturn same of 40 x 30 x 12 + 40 x 3 = 19,200#; and this would act at a point say 65 feet above the footing. Now the moment would be 192000 x 65 = 1,249,500 ft. lbs. Since the column centers are 30 ft. apart, the tension in one and compression in the other due to this overturning moment is - 1249500/30 = 41600#. This requires 41600/450 = 92 sq. in. of concrete in column. It is evident from an inspection of sheet #8 that there is this much to spare in all cases of loading, and this force will not be considered further.

5th

Stresses due to the change of temperature in concrete.

Tbis will need to be considered in a longitudinal direction only. Say the maximum change of temperature would be 140° and since the coefficient of expansion is .0000055 if an expansion joint be put in at every sixth panel there would be a maximum movement at that joint of ½ in.

6th

Horizontal thrust due to trains setting brakes while on dock.

The assumption is that when brakes are set to slipping .2 of the wheel load causes thrust in a longitudinal direction. From an inspection of the locomotive loading, given on page 6, it is evident that over a distance of 38 ft. 4 in., which is the distance over which there would occur a maximum load concentration, we would have a load of 310,000#. Then the thrust on the four tracks on the one side of the dock would be 310000 x 4 x .2 = 240,000#. The concrete sidewalks which are between the tracks and adhere to the I beams (I_4) would distribute this along the dock so it would be cared for by at least six panels, which is the number of panels between expansion joints. This force would require a sidewalk area of 240000/450 = 533 sq. in., and since the sidewalk on one-half of deck has an area of 1870 sq. in. we will not need to consider this further.

7th

Determination of the pressure on the floor of bin due to the weight of ore in same.

This pressure will be due to three forces; one which is the weight of the ore, one due to the impact of ore falling from cars, and one due to the horizontal force (R), (see page #10).

These forces are shown in the figure #5. A-B is the floor of the bin, O is the point at which the force is to be found, W is the pressure in #/sq. ft. of the column of ore directly above O and it is equal to the weight of a cubic foot of ore times the height (h) of the column, (f) is the force per cubic foot due to the falling of the ore, R is the pressure in #/sq. ft. which is given in table #3, and N is the resultant of these forces.

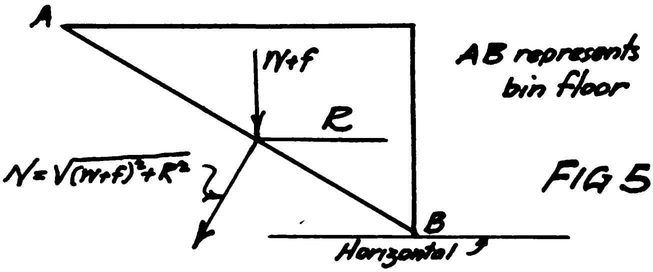


FIG 5

In regard to the force (f) the following discussion is necessary. Should a car of ore be emptied suddenly it would hit the floor of the bin with considerable impact, which would be conducted directly thru the light timber floor to the floor beams, and a calculation of this is therefore made so that provision may be made for this extra force. The well known theory of impulse, momentum, and of the laws of falling bodies is used. The time which might be taken for a car load of ore to fall from its position in a car to the bottom of the bin is a variable quantity and after consultation and observation I decided to

assume a time of two seconds. The area over which this falling mass of ore would be distributed is a quantity also subject to the judgment of the designer. An area of 120 sq. ft. will be assumed. A car load of ore is assumed to weigh 120,000#.

Now if,

m = mass = W/g

v = velocity of fall (ft./sec.)

F = force impact in lbs.

T = time of fall in seconds

W = weight of ore falling

h = height of fall in ft.

$g = 32$ = constant of gravity

and since,

momentum = impulse

$m\,v = F\,T$

and, $v^2 = 2gh$

we have, $m\sqrt{2gh} = FT$

and $F = m\sqrt{2gh/T} = W\sqrt{2h/gT} = 120.000\,\sqrt{2h}/2\sqrt{g}$

In table #5 the values of F in column #3 are calculated for each height of "h" as given in column #1. The effect of this falling mass of ore in #/sq. ft. is given in column #4 as f.

TABLE #5

(1) h'	(2) $\sqrt{2h}$	(3) F	(4) f=F/120	(1) h	(2) $\sqrt{2h}$	(3) F	(4) f=F/120
1	1.4	14,700	123	12	4.9	52,000	433
2	2.0	21,200	176	13	5.1	54,000	450
3	2.4	25,400	212	14	5.3	56,000	466
4	2.8	29,600	246	15	5.5	58,200	485
5	3.2	34,000	284	16	5.7	60,300	502
6	3.5	37,000	308	17	5.8	61,400	512
7	3.7	39,200	326	18	6.0	63,500	529
8	4.0	42,400	354	19	6.2	65,600	546
9	4.2	44,500	370	20	6.3	66,600	554
10	4.4	46,600	390	21	6.5	68,800	571
11	4.7	49,800	414	22	6.6	70,000	582
				23	6.8	72,000	600

TABLE #6

(1) h	(2) R	(3) W	(4) f	(5) (W+f)	(6) =N	(7) W+f/RM	(8) ϕ
1	47.5	160	123	283	287	5.2	80°
2	95.0	320	176	496	504	5.1	79°
3	143.0	480	212	692	706	4.9	79°
4	190.0	640	246	886	907	4.7	78°
5	238.0	800	284	1084	1100	4.6	77°
6	285.0	960	308	1268	1295	4.4	77°
7	333.0	1120	326	1446	1480	4.3	76°
8	381.0	1240	350	1637	1680	4.2	76°
9	429.0	1440	370	1990	1850	4.1	76°
10	475.0	1600	390	1990	2070	4.0	76°
11	525.0	1760	414	2174	2230	4.0	76°
12	572.0	1920	433	2253	2330	4.0	76°
13	620.0	2080	450	2530	2600	3.9	76°
14	667.0	2240	466	2706	2790	3.9	76°
15	715.0	2400	485	2885	2980	3.9	76°
16	763.0	2560	502	3062	3160	3.9	76°
17	810.0	2720	512	3232	3330	3.9	76°
18	858.0	2880	529	3409	3510	3.9	76°
19	905.0	3040	546	3586	3690	3.9	76°
20	954.0	3200	554	3754	3860	3.9	76°
21	1000.0	3660	554	3931	4050	3.9	76°
22	1035.0	3520	582	4102	4220	3.9	76°
23	1095.0	3680	600	4280	4410	3.9	76°

Table #6 shows the computations made in determining the value of N which is given in the figure #4. In column #1 is given the distance from the top of the bin to the point (O), Fig. #5. In column #2 is the value of R for that point as given in table #3. In column #3 is given the force which acts in a vertical direction and is equal to the weight of a column of the ore one foot square and of height (h). The values of (f) in column #4 are taken from table #5. The sum of W and f are given in column #5. In column #6 is given the value of N, which is the resultant pressure due to the ore and is equal to the square root of the sum of the squares of R and W + f. The tangent of the direction which N makes with the horizontal is given in column #7, and ϕ is given in column #8.

The floor beams and the floor will be designed for the value of N given in column #6.

DESIGN OF MEMBERS

(A) Front Wall of Bins.

Four alternative designs will be made and the cheapest one will be selected. The designs will be as follows: 1st- a circular front of steel plate; 2nd- a circular front of concrete; and 3rd- a flat front of reinforced concrete designed to act as a simple rectangular beam ex-

tending horizontally, and 4th- a flat front of reinforced concrete consisting of slabs and T beams extending horizontally.

1st. Consisting of bent strips of steel.

In this bent strips of steel plate are to be riveted together to form a semicircular section or barrel shaped front, the strips to run from a maximum thickness at the bottom to a minimum at the top, and the ends of the strips to be anchored in the concrete of the lateral partition walls of the bins. This method is advantageous in that the ore will slip bettwe on the steel plate than it would on the concrete and the weight of the large mass of concrete necessary to take care of the outward thrust of the ore is avoided. Moreover, the semicircular front affords more space in the bin than the square front.

The figure #6 shows roughly the style considered.

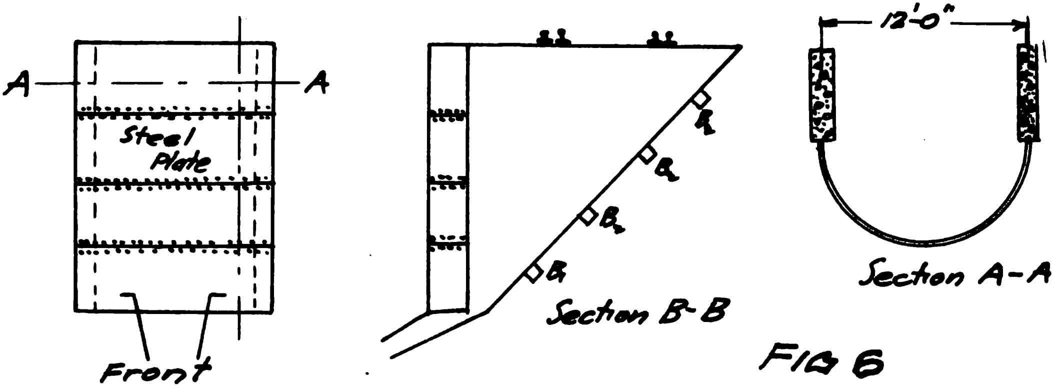


FIG 6

The ore pressure (R) (see under "Determination of Stresses") is assumed to act across the diameter of the section as in sewer design, and the maximum tensile stress is assumed to occur at a-a. Assuming that 6R is taken by each side, and using the allowable unit tensile stress as given in specifications of the American Bridge Company, the thickness of plate required is equal to 6R/12 x 16000, and is given in inches.

The table #7 gives this thickness in inches as computed in the form above.

TABLE #7

(1) h	(2) ins.	(1) h	(2) ins.	(1) h	(2) ins.
1	.001	9	.013	17	.025
2	.002	10	.015	18	.027
3	.004	11	.016	19	.028
4	.006	12	.018	20	.029
5	.007	13	.019	21	.031
6	.009	14	.021	22	.032
7	.010	15	.022	23	.034
8	.012	16	.024		

The thicknesses of plate given above are evidently not thick enough to stand the incidental strains which may come upon it; such as from the impact caused by ore sliding and falling from car and down the bin floor to front bin wall. The amount of this impact force and its effect is a difficult thing to calculate. However, I believe

that the following argument will at least approximate the truth.

The stresses due to these loads falling and striking the front of the bin suddenly would be practically double that due to the same load applied in the usual manner, that is gradually. The energy due to a gradual load applied from zero, when it first impinges, to a maximum is equal to 1/2Pd, where P is the maximum force and d is the deformation caused, and the energy of a sudden load is Pd' where d' is the deformation caused by the sudden load.

If Q is the equivalent static load required to produce the deformation d' the following is true:

$$1/2\ Qd' \quad Pd'$$

and, $Q = 2P$, $d' = 2d$, $d'/d = Q/P$

or a sudden load has twice the effect of a quiet load of the same amount.

Now the energy due to a force P impinging with a velocity V is equal to $Pv^2/2g$, where g is the force of gravity, and the distance of fall H necessary to acquire the velocity V is equal to $V^2/2g$, and the energy of fall

$HP \quad \frac{1}{2} Qd'$,

thence, $Q = 2HP/d' \;= P(2H/d)^{\frac{1}{2}}$ and $d = (2H/d)^{\frac{1}{2}}$

I will now determine the deformation due to the

quiet load P = 16000#, as was assumed in the calculation of the table #7, assuming that the stress in the semi-circular steel plate is distributed equally around the perimeter, as in hydraulics. The deformation due to this stress is equal to the length of the perimeter in inches multiplied by 16000 and divided by the coefficient of elasticity.

or, $d = 53,1416 \times 12 \times 16000/30000000 = .101$ in.

But, this 0.101 inches elongation is measured along the circumference of the bin, and the deformation we wish should be in the direction of the falling load, or as shown in the figure 6(a)

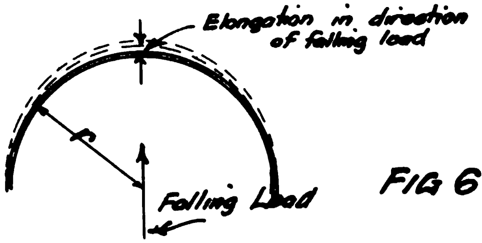


FIG 6(a)

The elongation desired is the change in radius due to an extension of the perimeter as given by the equation above.

since, $c = 2\pi r$, $dc = 2\pi dr$

and, $dr = \frac{dc}{2\pi} = \frac{0.202}{2\pi} = .032$ inch

In the equation, dc is the change in perimeter due to stress and it must be taken as twice 0.101 to account for

the entire circle.

Since $d'/d = Q/P$, we have the required thickness T' to stand the impact, is to the required thickness T° for the quiet load P as the equivalent load Q is to P, or $T'/T^\circ = Q/P$, and having $Q = P\ (2H/d)^{\frac{1}{2}}$ the following table #8 is computed, the value of H being taken as ½ the distance from the top down to the distance indicated by (h), because the effect of at least ½ the fall might be taken by the resistance offered while rolling down the bin. P = 16000#/sq. in., and (d) .032 ins., as computed above and called dr.

TABLE #8

(1) h	(2) H	(3) d	(4) P	(5) $Q=P(2H/d)^{\frac{1}{2}}$	(6) Q/P T'/T	(7) T'
7.6	3.8	.032	16000	248,000	15.5	.14
8.8	4.4	"	"	268,000	16.7	.22
11.0	5.5	"	"	290,000	18.1	.29
13.0	6.5	"	"	315,000	19.7	.38
14.7	7.3	"	"	342,000	21.3	.46
16.9	8.4	"	"	367,000	22.9	.55
18.7	9.3	"	"	390,000	24.3	.65
20.6	10.3	"	"	417,000	26.0	.75
22.7	11.3	"	"	430,000	26.8	.90

From this table it is seen that the plates required will vary in size from say 1/4 in. at the top to 7/8 in. at the bottom. The average thickness will be ½ of an inch, and the volume of steel required as computed from

this will be as follows:

$$12/2 \times \pi \times 20 \times 144 \times \frac{1}{2} = 25700 \text{ cu. in.}$$

Now the price of material in this front will be the volume of steel times the cost of the same per unit volume. Of course this is only a very approximate estimate, but it is sufficient to use in the comparisons of different styles of design.

Approximate cost 25700 x .0083 = $213.00. A sum of say $20.00 should be added for rivets and incidentals.

2nd.

Design of circular wall of concrete and steel.

This will be designed along principles similar to those used in the design of reinforced concrete sewer pipe, the concrete being used to give stiffness, as in a common tile pipe the clay is used, and circular steel being imbedded in same to take care of the bursting tensile strains due to the force R.

The figure #7 is a rough sketch of this.

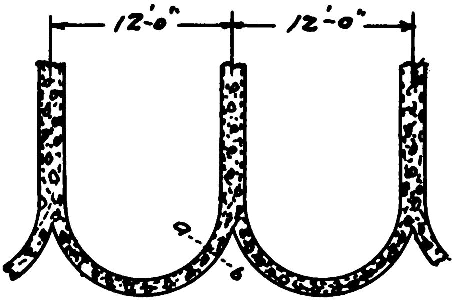


FIG 7
Horizontal Section thru Bins, showing reinforced concrete walls and front of bins.

The impact stresses assumed in the design of the steel front are probably larger than necessary, especially when designing for the more massive material, concrete. The steel plate, however, will need to be as thick as determined, for stiffness. It is evident that no such impact would occur when we consider that the capacity of the car is a small part of the capacity of the bin, and that the ore almost always falls slowly from the car, and even then the ore hits the bottom of the bin first and rolls or slides to the front with reduced velocity. Another fact which is of importance in this connection is that when the ore has a large distance of fall the bin is not already under the stress of a weight of ore, and when there is a load of ore in the bin the dropping ore can only have a short fall. For these reasons the impact will not be considered in this design, and should there actually be some impact due to ore suddenly sliding down the floor of the bin it would be largely dissipated in the mass of the concrete used.

I will make the thickness of the wall 5", and the amount of steel will be gauged by the outward thrust of the ore. This outward thrust is equal to (R) # / sq. ft., and the amount of steel required per foot of height, or the stress at a-b, see figure #7, is equal to 6R as half

goes to each side.

The table #9 shows the area of steel required in square inches to take care of the stress 6R using the assumed working stress of 16000 # /sq. in.

TABLE #9

(1) h	(2) sq.in.	(1) h	(2) sq.in.	(1) h	(2) sq.in.
1	.012	9	.156	17	.300
2	.024	10	.180	18	.325
3	.048	11	.190	19	.336
4	.072	12	.216	20	.350
5	.084	13	.228	21	.375
6	.098	14	.252	22	.385
7	.120	15	.264	23	.410
8	.144	16	.289		

Total area of steel required equals the sum of the above = 4.778 sq. ins.

Volume of steel = 4.778 x π x 5 x 12 = 902 cu. ins.

Cost of steel = 902 x .0083 = $7.43

Volume of concrete = 23 x 12 x 5π x 12 x 5 = 312,000 cu.ins.

Cost of concrete = 312000 x .000108 = $33.70

Cost of forms say, 5.00

Total cost $47.13

An extra feature will be required for designs #1 and #2 which would not be required with the square front; which will amount to an extra expense in these two cases.

This will now be considered. If one bin should

happen to be filled and the adjacent one empty there would be a thrust of R#/sq.ft. on the partition between the bins. This will be provided for by making the partition of concrete reinforced horizontally, and considered to act as a simple beam extending in the direction of the reinforcement and supported at back by the floor beams of the bin and at the front by the front wall of the bin. In the case where the front wall is flat, as in designs #3 and #4 this will be all right, but where the front is semi-circular it cannot take care of this force and a vertical beam will need to be placed in the wall to carry the loads to the girt I' above and to the floor beam B' below.

Figure #8 shows this beam as (a-b).

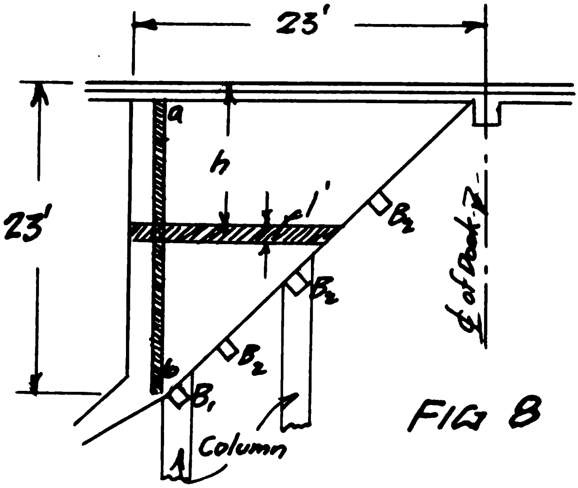


FIG 8

At any height (h) the force with which a section of the wall extending horizontally one foot wide would react upon this beam would equal R x 23 - h, divided by two, since half of this goes to the back of the bin. Then the equation of this shearing force is, $S = \frac{R \times (23-h)}{2}$

Now the total load on such a beam (a-b) would be the summation of S with respect to h between the limits of h 0, and h 23, or it would be the sum of the loads contributed by each horizontal strip one foot wide. R is a variable depending on h. See curve (R_3), where R is shown to be equal to 1000h/21. Inserting this value for R, we have for the summation of S

$$\sum S = \int_0^{23} 1000h/21 \times 2(23-h)\, dh$$
$$= \int_0^{23} 23000h/42\, dh - 1000h^2/42\, dh$$
$$= 23000\, h^2/84 - 1000h^3/126$$

Inserting the limits of h, $\sum S = 51,500\#$

This is the total load which the beams a-b, Fig. #8, would carry when one beam is full and the adjacent one is empty.

I will now determine the distance from the top of the beam of the point which is the center of pressure of these loads, $S = R \times \frac{(23-h)}{2}$ I will call this distance $\bar{X}$.

$$\sum (S \times h) / \sum S = \bar{X}$$

Now since $S = 23000h - 1000h^2/42$,

and, $\sum S \times h = 23000h/126 - 1000h^2/168$

and, $\sum S = 23000\ h^2/84 - 1000\ h^3/126$

we have, $\bar{X} = 183h - 5.95h^2/275 - 7.95h$

If $h = 23$, $\bar{X} = 1062/92 = 11.5$

This is the distance of the center of gravity of the forces on the beam (a-b) from the top of the beam, when one beam is full and the adjacent one is empty.

The reaction of this beam a-b, on the beam I' at the top and B at the bottom is equal to $\sum S \times \frac{11.5}{23.0}$ or

$P = \frac{51,500}{2} = 25,750$. This is true when the bin is full, which is the case we wish since the greatest moment will then occur. Now the moment at any cross section of the beam M_h, equals $(P \times h) - (\sum s \times \bar{X}h)$. In table #10 the moments at various points of the beam, which are at distance h from the top of the beam are computed from the formulae given above.

TABLE #10

(1)	(2)	(3)	(4)	(5)	(6)	(7)
h	xh	s_t	Pxh	S_t xh-x	M'#	M"#
5	3.4	5860	12850	9400	34500	415000
10	6.3	19450	257500	72000	185500	2230000
11	6.9	22600	283000	92500	190000	2280000
12	7.4	25700	309000	119000	190000	2280000
20	11.0	46000	515000	415000	100000	1200000

It may be seen from table #10 that the maximum moment is 2,280,000"#; and it occurs at a distance 12 ft. from the top of the beam.

Now from mechanics we have, M = f x I/C

where f = 16,000#/sq.in., and I/C = section modulus.

Then I/c = 2280000/16000 = 142

A structural steel section made up of 4 L 6" x 3½" x 13/16" and a plate 12" x 13/16" has a section modulus of 146.7, and may be used.

This has a cross-sectional area 37.99 sq. ins.

The volume of one beam would be 37.99 x 12 x 23 = 10450 cu. ins.

The cost would be 10450 x .0083 = $87.00

This gives a total for the round steel front of $247.00

and for the rounded front of concrete of $126.60

3rd

Front consisting of a massive slab of reinforced concrete, designed to act as a simple beam.

This design will be very brief, amounting to little more than an estimate of the cost.

The beam is 12' in length and it carries a uniform load of R#/sq.ft.

If M is the maximum bending moment and W is the uniformly distributed load, and I is the length of the beam we have the following:

$$M = WI^2/10$$

Let b be the width, assumed as one, and d be the depth of the beam.

From page 277 in Turneaure and Maurer's text we find for $f_c = 650$ and $fs = 16000$, and $n = 15$ that $M/bd = 105$ and the "percentage of steel" = .75%.

In table #11 in column #1 are given various values of h, in column #2 are the corresponding values of R taken from table #3, are the corresponding values of the maximum moment figured by the formula given above, in column #4 are given the corresponding values of 'd' as calculated from the data given above, and in column #5 are given the corresponding values of steel required.

TABLE #11

(1)	(2)	(3)	(4)	(5)
h	R	M	"d"	steel sq.in.
1	47.5	8200	2.0	0.2
4	190.0	32900	2.6	0.2
8	381.0	66000	5.3	0.5
12	572.0	99000	7.8	0.7
16	763.0	132000	10.5	0.9
19	905.0	156000	11.1	1.0
23	1095.0	189000	12.2	1.1
			51.5	4.6

The average thickness of the concrete is 51.5/7 = 7.5", and the average area of steel per foot height is 4.6/7 = 0.66".

Volume of concrete	7.5 x 144 x 23 x 12 = 298000 cu. ins.	
Cost " "	0.000108 x 298000 =	$32.30
Volume of steel	0.66 x 23 x 144 = 2180 cu. ins.	
Cost " "	0.0083 x 2180 =	18.20
	Incidentals and stirrups =	5.00

	Total cost =	$55.50

4th.

Consisting of square front of reinforced concrete in the form of slabs and T beams.

The ribs or T beams will extend horizontally across the front, being anchored to the partition walls of the bins. They will be designed to carry 2000#, 3000# and 4000# per linear foot, and will be spaced so that they will receive these loads. The slabs will be reinforced vertically. The thickness of slab and percentage of reinforcement will be calculated first.

It will be understood that these loads are simply assumed for the purpose of design, and the beams are so spaced at the front of the bin that they will receive the loads for which they are designed.

In table #12 the thickness of slab and amount of steel reinforcement are given. The value of R is of course taken from table #3, and the thickness of slab 'd' and the area of steel A, and also 'c' the distance of steel in slab from face of same are taken from "Engineer's Hand Book", published by the American Steel and Wire Company. The triangular mesh concrete reinforcement will be used which is produced by this company.

TABLE #12

(1)	(2)	(3)	(4)	(5)	
(h)	(R)	(d)	(A)	(c)	
1	47.5	3.0	0.16	3/4	The total load per foot here is 475.5#
2	95.0	"	"	"	
3	143.0	"	"	"	
4	190.0	"	"	"	
5	238.0	"	"	"	The total load is 2141.0#, taken by beam B
6	285.0	"	"	"	
7	333.0	"	"	"	
8	381.0	"	"	"	
9	429.0	4.0	"	"	
10	475.0	"	"	"	
11	525.0	"	"	"	The total load is 3099.0, and is taken by beam B
12	572.0	"	"	"	
13	620.0	"	"	"	
14	667.0	"	"	"	
15	715.0	"	"	"	
16	763.0	"	"	"	The total load is 4090.0, and is taken by beam B
17	810.0	"	"	"	
18	858.0	"	"	"	
19	905.0	4.5	"	"	
20	954.0	"	"	"	

The load on the first four feet down from the top of the wall is taken care of by the arch of concrete between the girts I, at the top of each partition wall, (see sheet #9). A few calculations which the designer has made show that this arch is plenty strong enough to carry this load. The other parts of the wall are taken care of by the beams which are spaced to carry their respective loads.

In table #13 three beams are designed as T beams to take the uniformly distributed loads of 2000#, 3000# and 4000#, which loads are assumed for the purpose of design, and the beams will be spaced so as to get these loads. These beams will be 12 feet long. The widths of web are assumed after making several trials, and are given in column #4 of the table. The shear at every foot of each beam is given in column #3, and the trial depths in column #5 are made such that the area of the cross-section will take care of the shear. These depths are very small and the depths used are given in column #7. In column #2 the moments are given for each foot as figured for each foot. The depth in column #7 and the area of steel in column #8 are calculated by trial so that the area of steel times 16000 times the distance from center

of steel to center of slab equals the bending moment at each foot. Column #6 gives the area of the slab which may be considered as carrying the compression, and is equal to three times column #4 times the thickness of the slab at the location of the beam. The area of steel in column #8 is column #2 divided by (16000 x 7/8 of column #7). Column #9 equals column #8 times 16000, and equals the tensile stress in the steel and the compressive stress to be cared for by the concrete. Column #10 equals column #6 times $f_c/2$. (f_c is taken since 1/2 the maximum equals the average). Since column #9 is less than column #10 there is plenty of compressive area in the part of the slab which is considered to act in compression. It can be seen from the table that the assumed depth is as great as the required depth, so that in regard to shear the beam is safe. However, 1/4 inch stirrups will be used for stiffness and safety and will be spaced as shown on the detail plan #14. All details of these beams are shown on sheet #14 of the plans.

TABLE #13

Beam B_3 (see sheet of plans #9)

(1) Dist. from end of beam	(2) moment (ins.#s)	(3) shear (#)	(4) b ins.	(5) d req'd (ins)	(6) area of slab (ins)	(7) d used (ins)	(8) A area of steel (ins)	(9) C=T (#)	(10) C' (#)
1	132000	10000	6"	13	54	13	0.9	22500	23400
2	240000	8000	"	10	"	13	1.3	"	"
3	324000	6000	"	8	"	16	1.4	"	"
4	384000	4000	"	5	"	19	1.4	"	"
5	420000	2000	"	2	"	21	1.4	"	"
6	432000	000	"	0	"	24	1.3	"	"
7	420000	2000	"	2	"	21	1.4	"	"
8	384000	4000	"	5	"	19	1.4	"	"
9	324000	6000	"	8	"	16	1.4	"	"
10	240000	8000	"	10	"	13	1.3	"	"
11	132000	10000	"	13	"	13	0.9	"	"

Beam B_4 (see sheet #9 of plans)

1	198000	15000	8"	14	96	14	1.4		31200
2	360000	12000	"	11	"	14	1.8		
3	486000	9000	"	8	"	18	1.8		
4	576000	6000	"	6	"	22	1.8		
5	630000	3000	"	4	"	26	1.8		
6	648000	0000	"	0	"	26	1.8	29000	
7	630000	3000	"	4	"	26	1.8		
8	576000	6000	"	6	"	22	1.8		
9	486000	9000	"	8	"	18	1.8		
10	360000	12000	"	11	"	14	1.8		
11	198000	15000	"	14	"	14	1.4		

Beam B (see sheet #9 of plans)

TABLE #13 (continued)

(same column headings as on page 40)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	264000	20000	10	15	150	15	1.9		50000
2	480000	16000	"	12	"	18	2.1		
3	648000	12000	"	9	"	23	2.1		
4	768000	8000	"	6	"	28	2.0		
5	840000	4000	"	4	"	28	2.1		
6	864000	000	"	0	"	28	2.2	35000	
7	840000	4000	"	4	"	28	2.1		
8	768000	8000	"	6	"	28	2.0		
9	648000	12000	"	9	"	23	2.1		
10	480000	16000	"	12	"	18	2.1		
11	264000	20000	"	15	"	15	1.9		

The part of these beams between the dashed lines which are indicated on sheet #14 is to be formed on the ground, and when raised to position to be filled in around the 12"-31.5# short piece of I beam which is riveted to member I. That the rivets supplied are strong enough is shown in the following table.

TABLE #14

(1)	(2)	(3)	(4)
Beam #	No. of rivets	Total strength	Maximum shear #
3	6-5/8 in.	22,000	20000
4	6-3/4 "	32,000	30000
5	8-3/4 "	42,500	40000

Since in every case the strength exceeds the shear the rivets are safe.

Cost of the 4th design is as follows -

Slab

Volume of concrete - 4.4" x 23x12x144 175,000 cu.in.
Cost " " - 0.000108 x 175000 = 18.95

Volume of steel - 0.16x12x23x12 530 cu.in.
Cost " " - 0.0083 x 530 = 4.40

$23.35

Beam #5

Volume of concrete - 15x10x144 21,600 cu.in.
Cost " " - 0.000108 x 216000 = 2.34

Volume of steel - 2.2 x 144 317
Cost " " - 0.0083 x 317 = 2.63

$4.97

Beam #4

Volume of concrete - 13x8x144 15,000 cu.in.
Cost " " - 0.000108 x 15000 = 1.62

Volume of steel - 1.8 x 144 260 cu.in.
Cost " " - 0.0083 x 260 = 2.16

$3.78

Beam #3

Volume of concrete - 12x8x144 10,400 cu.in.
Cost " " - 0.000108 x 10400 = 1.12

Volume of steel - 1.4 x 144 202 cu. in.
Cost " " - 0.0083 x 202 = 1.68

$2.80

Total cost = $38.60

This form of front being the cheapest of the four styles, it will be used.

B Design of lateral walls of bins.

The lateral or partition walls of bins will be designed to take the train loads from the beams B, (see sheet #9), and to take the loads R#/sq.ft. when one bin is full and the adjacent one is empty. Practice has shown that a concrete wall one foot thick will serve well here. I will make the wall one foot thick and will put in sufficient horizontal steel reinforcement to take the moment due to the force mentioned above. I will assume the beam to act as a series of simple horizontal rectangular beams, supported at one end by the floor beams and at the other by the front wall of the bin. The stress might be toward one bin or it might be toward the other, and the reinforcement is put in both sides of the wall so that this reversible stress may be cared for.

Now the front beams B_3, B_4, and B_5 (see sheet #9) have outward thrusts when the bins are as full as shown in column 4 of table #13. This is transmitted to beam I by the rivets shown on sheet #14 and in table #11. I_2 transmits this stress to the floor beams by rods extending thru the walls, one half on each side. The stress (R) acts equally on front wall of bin and on floor of bin, as shown in the determination of the stresses in the floor beams; and it acts in opposite directions. Thus the stress in the rods connecting these two members helds them in

equilibrium.

Since the rods mentioned above extend half on one side of the wall and half on the other, and since when one bin is full and the adjacent one is empty only one half of the total shear shown in table #14 can act, we can assume that only half the rods are acting at such a time and that the other half may then act to carry the bending moment caused by the fact that one bin is empty and the adjacent one filled.

Table #15 shows the size and area of these rods.

TABLE #15

(1) member #	(2) stress #	(3) area of steel required	(4) rods	(5) area of same used
B	20000	1.25	4-1/2x2	1.56
B	30000	1.87	4-5/8x2	2.46
B	40000	2.50	5-1/2 & 4-3/8x2	2.86

The area of steel in rods of table #15 is the (A') of table #14.

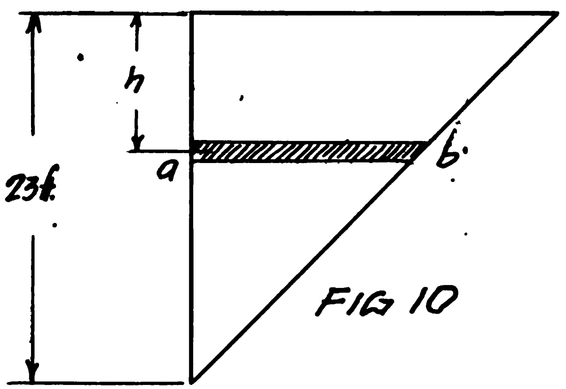


FIG 10

In the figure #10 is shown a section of the wall, a-b, at a distance (h) from the top of the bin. From the curve (R_3) we know that $R = 1000h/21$ and (R) = the uniformly distributed load on the strip of wall a-b. Since the bin floor makes an angle of 45° with the vertical it is evident that the length of beam considered at any time $I = 23-h$. Now we assume that the bending moment $M = WI^2/10$, where $W = R$. Therefore $M = (1000h/10x21)\ (23-h)^2 = (2515h-218h^2+ 4.7h^3)$ 12 in #. In table #15 the bending moment at each distance h of the assumed beam a-b from the top of the bin is given in column #2 as it was calculated from the above formula for moment. In column #3 is given the required area of steel for each face of the wall. A is taken equal to $M/16000j$, where $(j) = 10"$, the distance between the reinforcement in the two walls. In column #4 is the area of steel in the rods which connect the beams

I_2 and I_3, which may be considered as helping to carry the moment represented in column #2.

In column #5 is the area of steel remaining to be supplied, or it is column #3 minus column #4. Column #6 represents the amount of this which is supplied by "triangular mesh wire". The mesh to be used is the style #39 of the American Steel and Wire Company. The rest of the area shown in column #5 is made up of steel rods which area sized in column #8. The length of these rods is given in column #9 and their area is given in column #7.

TABLE #16

(1) h	(2) moment"#	(3) A	(4) A'	(5) A-A'	(6) A"	(7) A"'	(8) size rod	(9) length rod
1	27,500	0.18	0.00	0.18				
2	50,500	0.31	0.00	0.31				
3	68,600	0.42	0.00	0.42	0.325	0.09	2-1/4	7'
4	82,600	0.51	0.00	0.51	0.325	0.19	1-1/2	6'
5	92,600	0.57	0.00	0.57	0.325	0.24	2-3/8	5'
6	99,000	0.62	0.20	0.42	0.325	0.09	1-3/8	5'
7	103,000	0.66	0.20	0.46	0.325	0.13	1-3/8	4'
8	101,500	0.63	0.20	0.43	0.325	0.10	1-3/8	4'
9	101,000	0.63	0.20	0.43	0.325	0.10	1-3/8	3'
10	96,400	0.60	0.00	0.60	0.325	0.27	2-7/8	3'
11	90,000	0.56	0.15	0.41	0.325	0.08	1-3/8	3'
12	83,200	0.52	0.31	0.21	0.325			
13	74,500	0.46	0.31	0.15				
14	65,000	0.40	0.31	0.09				
15	54,800	0.34	0.20	0.14				
16	44,900	0.28	0.21	0.07				
17	35,000	0.22	0.21	0.01				
18	25,700	0.15	0.21	0.00				
19	17,400	0.10	0.60	0.00				
20	10,300	0.06	0.60	0.00				
21	4,800	0.03		0.03				
22	1,400	0.01		0.01				

Total amount - 8.26 sq. ins. of steel

Cost of wall is as follows:

Volume of concrete	- 23x23x1728x1/2 = 437,213 cu.ins.		
Cost " "	- 437,213 x 0.000108	=	$47.20
Volume of steel	- 695.4 cu. ins.		
Cost " "	- 695.0 x 0.0083	=	5.77

	Total cost		$52.97

We now design a structural steel member to be buried in the top of this concrete wall and to distribute the load to the wall from the beams I_4. The member to be designed will be an I beam and will be called I_1, (see sheet #9).

Try a beam which is about 5" wide. It is fair to assume that such a beam imbedded in an upright position along the center of breadth of the 12" wall will with its top and bottom flange and adhesion on the web have a total bearing which might be considered equivalent to the bearing area of a beam 10" wide.

Now the maximum double load from the ends of abutting beam I is 100,000#, and the bearing area of such a beam per linear foot is 120 sq. ins., and at 450#/sq.in. the concrete would react on the beam with a force of 120 x 450 54,000#/lin. ft.

The linear feet of wall over which the load may be

considered to act is 100000/54000 = 1.85'.

The figure #11 shows this.

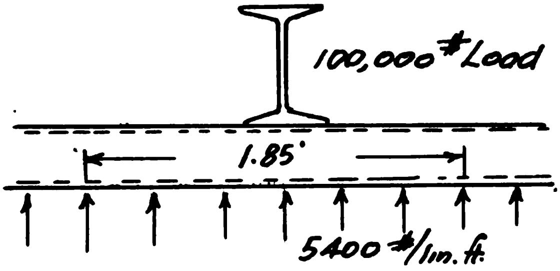


FIG 11

Now if the concrete wall is considered to act upward on the beam as a uniformly distributed load, we will have the bending moment in the beam of 1/4-Wl or 54000 x 1.85 x 12/4 = 270,000"#.

M/s I/c 270000/16000 17"

A 9"-21# I beam will be used here as its I/c 18.9

The cost of this beam is 6.31 (its area) x29x12x0.0083 $18.20

A steel member having the same section as the above designed I beam will be put in the front of the wall I_2 (see sheet #9), and also at the bottom of the wall. Most of the reinforcing rods in the wall will be attached to these as will also the floor beams and the beams at the front of the bin. Thus these will act to distribute the stress from the front and bottom thruout the walls of the bin and among the reinforcing rods of the walls. They

will also act to stiffen the walls and provide a frame to which the forms may be attached during construction. They will have together a length of 46.5', and their area being 6.31 sq. ins., their volume is 3521 cu. ins.

Their cost is $29.24 per bin.

C Design of wooden floor of bin.

The wooden floor will consist of planks which will rest on the floor beams, and will extend in the direction in which the ore runs. It will be made of hard maple, the strength of which is taken from Cambria Handbook. We will have four spans and a cantilever of the floor planking to consider. The arrangement is shown in the figure #12.

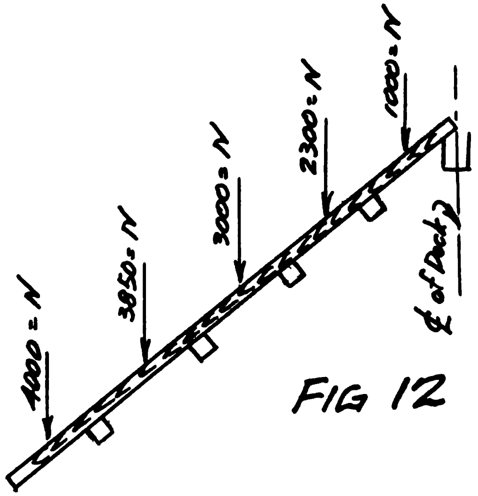


FIG 12

Table #16 shows how the bending moment is computed.

Span	Average load per ft. on span (N)	Span Length	Moment
Cantilever a to chute	4000	12.5	28,400"#
simple beam a to b	3850	25.0	112,000"#
simple beam b to c	3000	25.0	87,000"#
simple beam c to d	2300	49.0	131,000"#
simple beam d to center	1000	144.0	166,000"#

The thickness of the floor must be designed to care for the maximum moment, 166,000"#.

As before, $M = sI/c$

Now $I/c = bd^2/6$ for rectangular beams

Then $d^2 = 6M/17s$

I assume (s), the maximum allowable fibre stress for wood, to be 1200#/sq.in., and (a) then $= \sqrt{6x166000/17x1200} = 7"$.

The maple floor will be made therefore 7" thick.

See sheet #9 for details as designed.

D Design of floor beams.

These beams will span the 12' panels between the columns, and will support the wood bin floor already de-

signed. From the several forms of beam which I have designed I select the trussed style as it is much the cheapest. The plan is to design beams which will sustain certain loads per linear foot and then to space them so that the floor load will all be cared for.

(a) Below is the design of a rectangular beam to take a load of 20,000# per linear foot.

Now the maximum moment equals $wl^2/10$ = 20000x144x12/10

M = load in # per sq. ins.

l = span in ft.

V = Wl/2 20,000x12/2 = 120,000#

V = shear in #

V/v = 120000/120 1000" = bd, where b = breadth, d = depth.

Table #17 shows the possible combinations of width and depth with the corresponding required areas of steel.

TABLE #17

(1) d"	(2) b"	(3) R M/bd	(4) f	(5) p(%)	(6) A(")
40	25.0	86.5	550	0.60	6.0 (1)
50	20.0	69.0	495	0.49	4.9 (2)
55	18.0	63.2	470	0.47	4.7 (3)
60	16.6	58.0	440	0.41	4.1 (4)
65	15.4	53.0	420	0.37	3.7 (5)

I will try #3 because I am afraid that a narrower beam would be apt to buckle and the wider beams require

more steel. The value of R given in column #3 is taken from table III, page 277, Turneaure and Maruer's text book. The values of f_c and p are also taken from this table.

Cost of this beam is given below.

Volume of concrete	- 144x1000 = 144000 cu. ins.		
Cost " "	- 144000x0.000108	=	$15.60
Volume of steel	- 144 x 4.7 = 676 cu.ins.		
Cost " "	- 676 x 0.0083	=	5.63
	Total cost		$21.23

(b') Design of a trussed beam of concrete and steel to take the uniform load of 2,000#/linear ft., for a span of 12'. The compression members will be designed of concrete and the tensile members of steel imbedded in the concrete. The figure #13 is a force diagram of the beam to be designed. I decided to make the beam of the dimensions shown, after a great deal of consideration. On figure #13 the stresses determined below are shown.

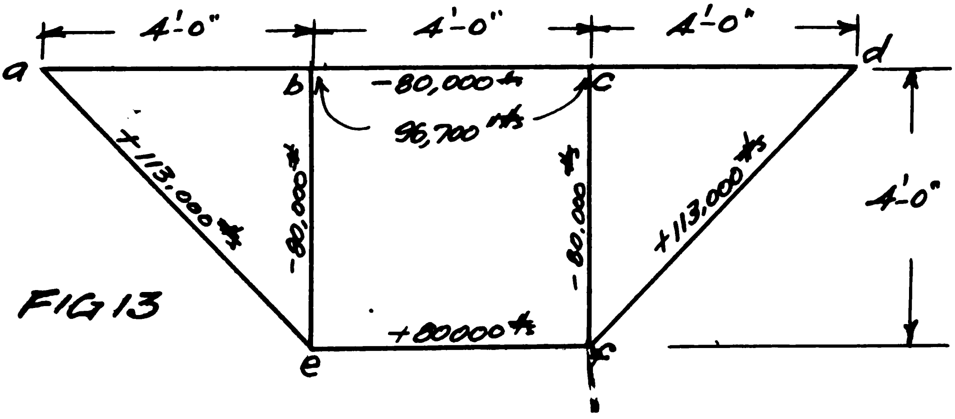


FIG 13

I will let members a-b, b-c and c-d act as simple beams of 4' span. The bending moment for each will be $M=Wl/10 =$ $20,000 \times 4 \times 12/10 = 96,700$"#

The shear for each is $Wl/2 = 20000 \times 4/2 = 40,000\# = S$

The stress in b-c and c-f = 2s = 80,000# compressive.

The stress in a-b-c-d = stress in e-f = 80,000/cos 45 = 113,000#

Steel in a-e and d-f = 113000/16000 = 7"□ Use 4-1-1/2" rods.

Steel in e-f = 80,000/16000 = 5"□ Use 4-1-1/4" rods.

Concrete in a-b-c-d = 80000/450 = 178"□ Make it 12" x 15"

Concrete in b-e and c-f = 80,000/450 = 178"□ Make it 12" x 15"

In b-e and c-f I will use 4-3/4" rods at the corners.

Now the compression in a-b, b-c and c-d in 80000#, and the moment is 96,700"#.

Assuming the member to act as a column eccentrically loaded, and the eccentricity, e, = M/N = 96,700/80,000 = 1.2"

$e/d = 1.2/15 = 0.08$

$M/bd^2 f_c = 96000/12x15 \times 650 = 0.055.$

Now entering the diagram on page 287 of Turneaure and Maurer with this value of e/d and $M/bd^2 f_c$ I find that 0.3% steel is required in the top and bottom of the member.

Then the area of steel in each face, A = 12x15x0.003 = 0.5"

Cost is as follows.

Volume of concrete in top chord - 144x15x12 =	25,900
" " " " struts - 2x36x15x12 =	13,000
" " " " bottom chord -6x12x144 =	2,060

	40,900 cu.in

Cost - 40900 x 0.000108 = $4.40

Volume of steel - 5x48 7x67 1x144 .88x36 = 884 cu. ins.

Cost " " - 884 x 0.0083= 7.30

Total approximate cost - $11.70

(b) Design of a trussed beam of concrete and steel to take a load of 10,000#/linear ft., on a span of 12'.

The stresses and areas of compression members will be one half those of the corresponding members in the beam designed to carry twice the linear load.

The dimensions and stresses as calculated are shown in figure #14.

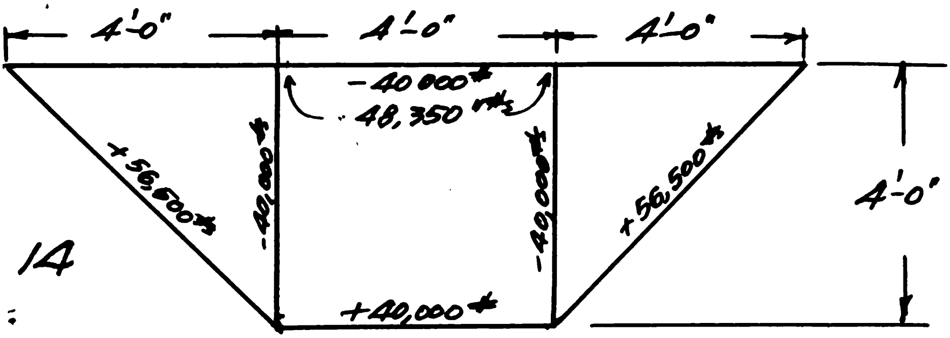


FIG 14

M, the moment in a-b, b-c and c-d, is 48,350"#

N, the compression in the same, is 40,000#

$e = M/N \quad 48{,}350/40{,}000 = 1.2$

$e/d = 1.2/10 = 0.12$

$M/bd\,f = 0.082$

Now entering to page 287 Turneaure and Maurer with e/d and M/bd f as an argument I find that 0.6% steel is required in each face.

The amount of steel is therefore A = 90 x 0.06 = 5"□

The size of members was figured as for the beam to carry 20000#/lin.ft., and the details of both beams as designed are given on sheet #15 of the blueprints.

The cost of this beam is as follows:

Volume of concrete in top chord - 9x10x216 = 19450 cu.in.
Cost " " " " " - 19450x0.000108 = 2.10
Volume of concrete in struts - 5x9x180 = 8110 cu.ins.
Cost of " " " - 8110x0.000108 = .878
$2.98

Total volume of steel 12.5x48 3.5x67 1x144 88x36 = 420.7
Cost " " 420.7 x 0.0083 =$3.50
Total cost of beam $6.48

E Design of center rib of dock, to act as floor beam and to give general stiffness to the dock.

This will be designed to carry a load of 10000#/lin.ft.

The spacing of the other beams is such that this load will occur. The beam will extend along the top of the center line of the dock, the top of it being used for a floor of the deck of the dock, and the maple floor will rest in a notch in the side of the beam. A section of this is shown on sheet #9. The beam will be made with sufficient area to carry the shear and enough steel will be put in to carry the moment.

$$M = W1^2/10 = 1,728,000''\#$$
$$S = W1/2 = 60,000\ \#$$

Area for shear = 60,000/120 = 500.0"

I will make it 15" x 33" = 505"

$R = M/bd^2 = 104.$

For $R = 104$, $f_s = 16,000$ and $f_c = 650$ we have from table #3 (Turneaure and Maurer) that p = 0.75%. Then A = 0.075 x 5000 = 3.75".

I will use 5-1" rods. The length of the rods are given by the following formula:

$$x_n = 1/(A\sqrt{a_1\ a_2 ------ a_n})$$

where x_n = length of the nth, rod in order of length.

1 = length of span 12'

A = total area of steel at center of beam.

$a_1\ a_2$-- a^n = area of each rod up to the nth rod.

Now 1/A = 6.2 = length of 5 rods, and applying the formula 7.8 is the length of the 3 rods and 9.5' is the length of 2 rods.

F Design of columns.

This was treated on page #12, under the topic, "Stress in columns supporting bins due to train load, ore load, and weight of structure."

G Design of footing to columns.

The weight of the column is say 1550#/lin.ft.

Say column is 48' high and weight of same is 75,000#
Maximum load on from dock is 940,190
---------
Total load to be carried by column 1010,000#

The assumed load per pile is 20 tons

No. of piles is 1010000/40000 = 28

The piles under each column will be arranged as shown on sheet #12. 8" x 10" caps will be used on the piles as shown in the drawing mentioned. These will act by virtue of their tensile strength to prevent the piles from spreading, and will act as shear reinforcement in the base of the footing thus allowing the value of (v) of 120#/" when figuring the shear on the section of the column footing. The depth of concrete required for shear over the outer row of piles at broad side of column is equal to the total upward pressure due to those piles divided by the length of footing in inches at the section and the allowable shearing stress (120). That is, it is equal to

40000x3/17x12x120 = 5". In the same way the depth over the next row of piles extending laterally in regard to the dock is equal to 40000x7/1x12x120 = 12", and across the two short rows of piles which are not directly under the column the depth of concrete required is calculated in the same way and found to be 8" and 20". The batter on the footing is made such as to insure this area for shear. The arrangement of piles and design of footing is shown on sheet #12.

H Design of steel imbedded in concrete column and supporting end of floor beams.

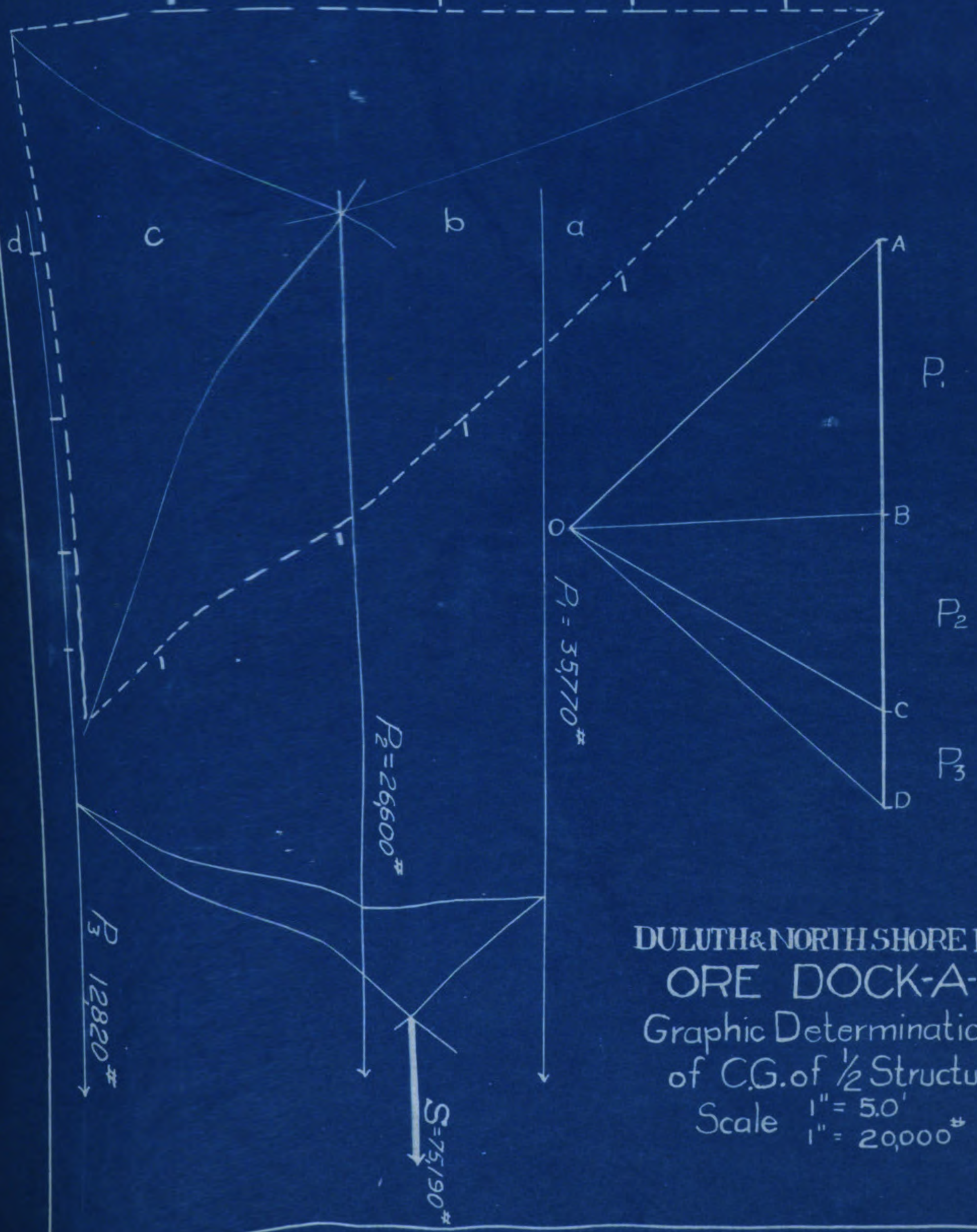
The total weight from the two abutting ends of two floor beams is 240,000# for beam B and 120,000# from beam B. In each case a plate (A) having an area of 144 sq. ins. will receive the load from the steel of the beam. This plate will itself act in bearing on the column to the amount of 144 x 450#/" = 64,600#. The resulting stress to be cared for by rods extending up into the wall and connecting on to member (I^2) is 175,400# for the beam B^2 and 5,540# for the beam B'. The area of steel rods required to support B is 175400/16000 = 11.0 sq. ins. That required to support B is 5540/16000 = 0.35". This required area is made up in the bolts shown in the position considered on sheet #15.

Accepted

W S Kinne

Asst Prof. Structural Engineering.

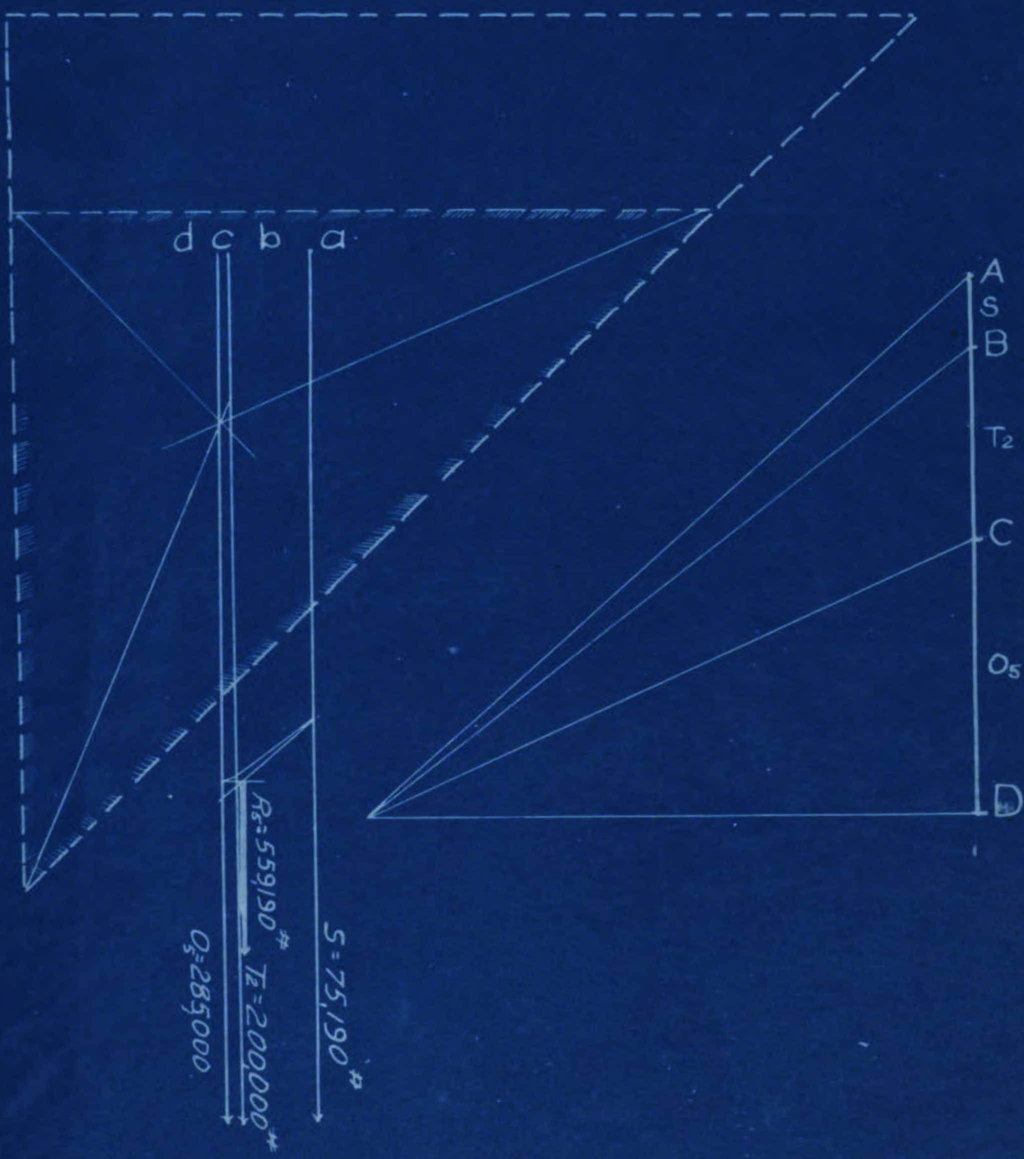
SHEET 1



DULUTH & NORTH SHORE RY

ORE DOCK-A-1

Graphic Determination
of C.G. of ½ Structure
Scale 1" = 5.0'
1" = 20,000#

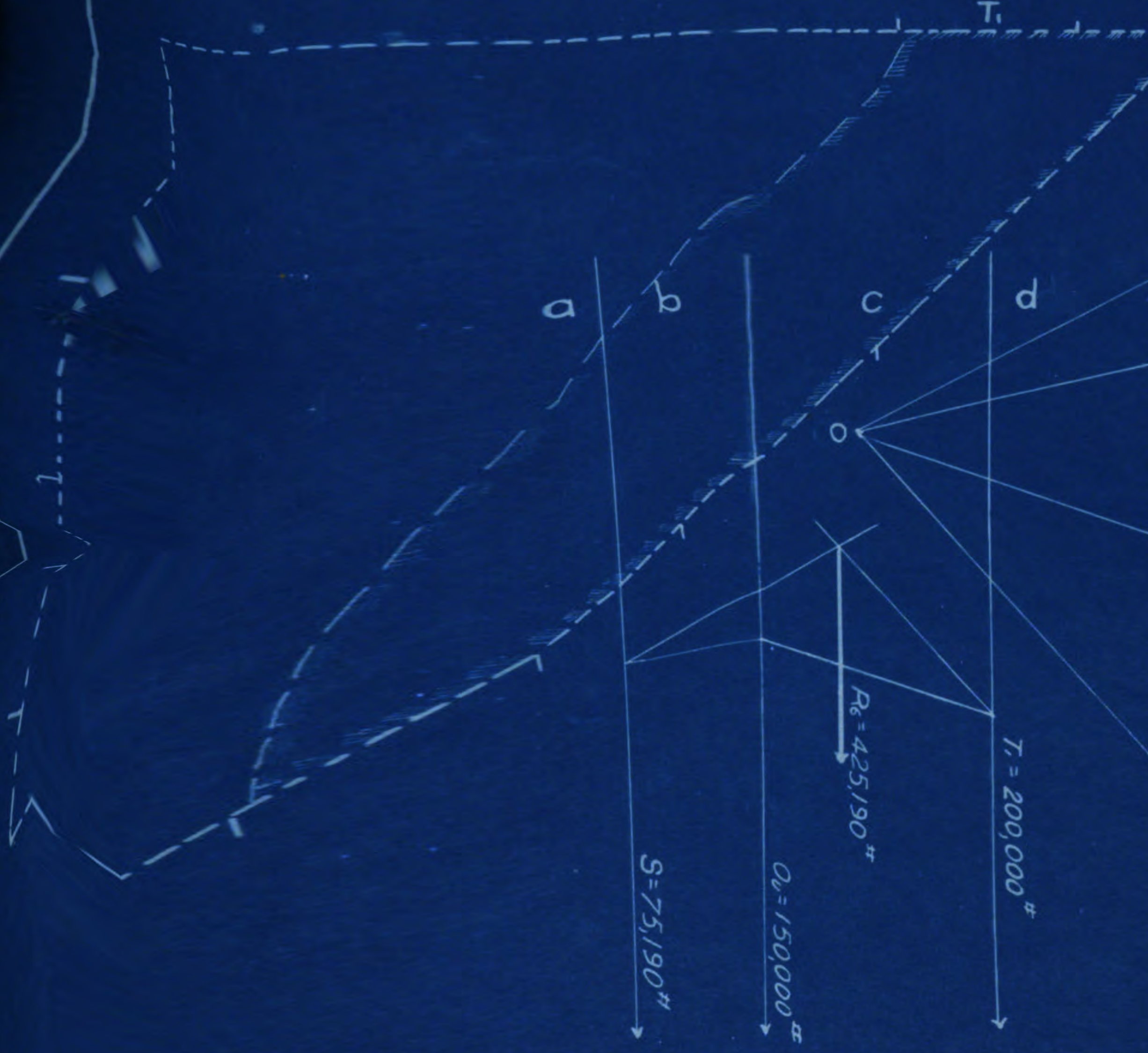


DULUTH & NORTH SHORE RY.

ORE DOCK-A-1

Graphic Determination
of Resultant of T_2 S & O_5

Scale { 1" = 5.0'
1" = 200,000#



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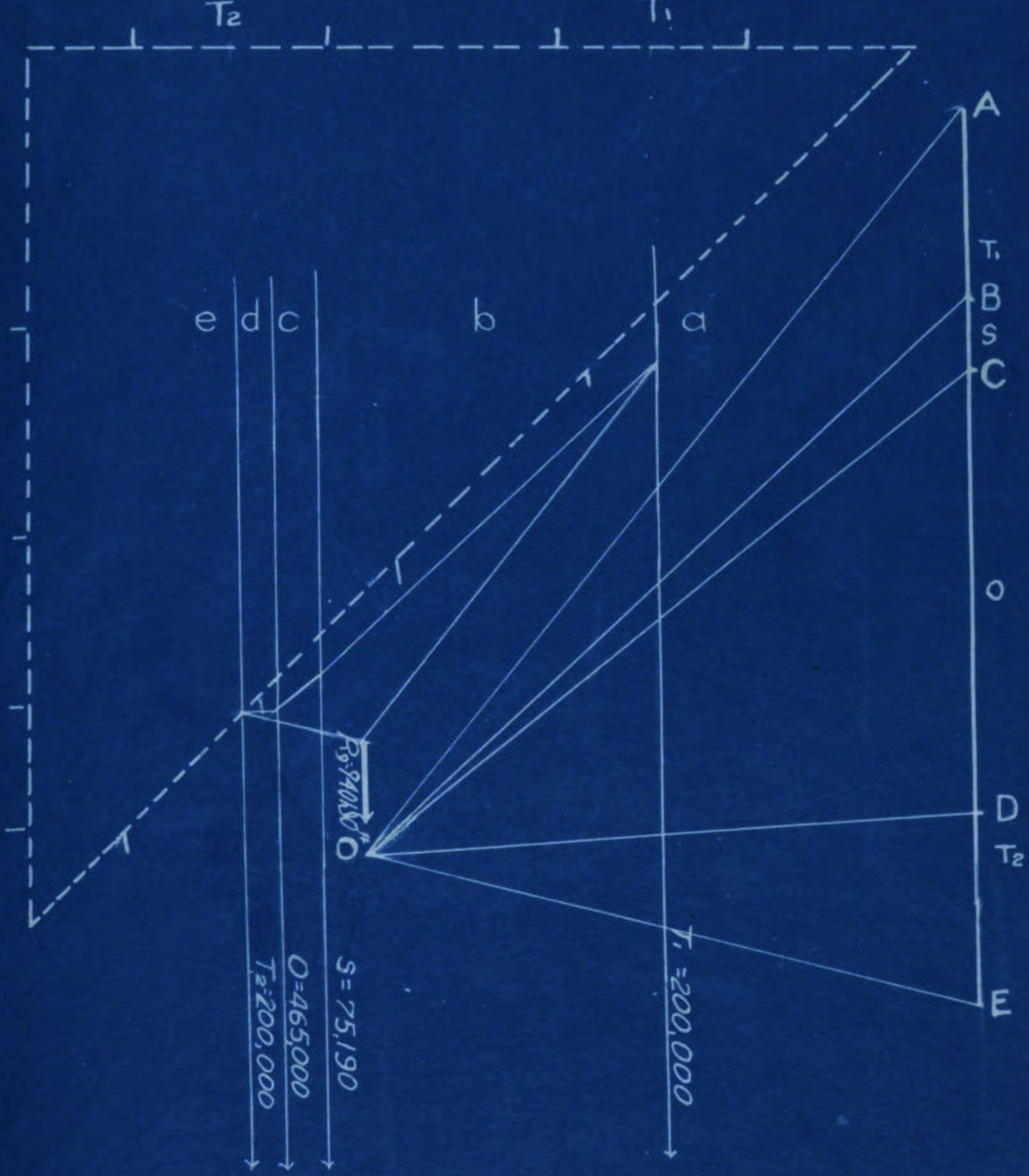
Graphic Det

of Resultant

Scale {1"
1"



DULUTH & NORTH SHORE RY

ORE DOCK-A-1

Graphic Determination
of Resultant of T_1 T_2 S & O

Scale { 1" = 5.0'
1" = 200,000#